6

Physical and Mechanical Properties of Rocks

6·1. Introduction

The performance of rock, under a particular condition, depends upon physical and mechanical properties of rock materials. The physical properties may be known as index properties, which describe the rock materials and help in classifying them. The mechanical properties may be known as strength properties and they give an information about the performance of rock materials, when subjected to a particular loading system.

A rock material is an aggregate of mineral particles. These individual mineral crystals and grains are not homogeneous, isotropic and perfectly elastic bodies due to sequence of formation of rock masses. The crystal hardness are influenced by various physical and mechanical properties, such as cohesion, brittleness, tensile strength etc., at the same time due to an orientation of cleavage planes. Direction of application of load is also an important factor while determining the rock properties. When acted upon by high stresses for a long period of time mineral crystals and crystal grains show time-dependent strain which is known as creep deformation or flow, and which causes an internal rearrangement of atomic structure of mineral. It can be summarised that the mechanical properties of a rock mass depends on the following factors.

(*a*) The mechanical properties of the individual elements constituting the system.

(*b*) The sliding friction along the planes of weakness.

(*c*) The configuration of the system with respect to direction of loading.

(*d*) The induced stress condition inside the mass.

The property of rock depends on the size of rock mass also. On a megascale the structural properties of the rock mass, such as bedding, cleavage plances etc. are important, while on a macroscopic scale, mechanical properties are important whereas on a microcopic scale the physical properties may be important.

6·2. Physical and Mechanical Properties

Physical properties, which are of interest in rock mechanics are : Index

(i) Porosity

(ii) Density

(iii) Moisture content

(iv) Degree of saturation

(v) Coefficient of permeability

(vi) Electrical properties

(vii) Thermal properties

(viii) Swelling

(ix) Anisotropy

(x) Durability.

Mechanical properties which are of interest in rock mechanics are : Strength properties.

(i) Strength

(ii) Deformability

(iii) Elasticity and Plasticity

(iv) Hardness.

Physical Properties

These are the properties of the material inheritated at the time of formation. These help in classifying a rock material. These are also called index properties of a rock specimen.

6·3. Porosity

Porosity identifies the relative proportion of solids and voids.

The porosity of a rock sample is defined as the ratio of the volume of voids to the total volume of the sample

i.e.
$$n=\frac{V_v}{V}\times 100 \qquad \ldots(6\cdot1)$$

where

n is the porosity generally expressed in percentage ;

V_v is the volume of void *i.e.* volume of air as well as water present in the pore spaces ;

V is the total volume of the rock sample.

If the sample is completely dry, voids will contain only air whereas in case of a fully saturated sample, voids will contain only liquid such as water or oil. Porosity depends upon the shape of mineral grains, their grading, orientation and the degree of compaction and cementation. When the rock forming particles are of different sizes, the space created by bigger particles will be filled by finer particles and thus, the rock will be having dense compaction resulting into a lesser porosity. On the other hand, if the grains are of uniform size, the spaces inside the mass will be large and thus, the rock will be having a higher porosity value. Due to weathering in the rock mass, and microfractures, the porosity of the rock may be higher. Porosity generally decreases with the age of the rock. It decreases with the depth also. Because at a greater depth, a rock is subjected to a higher pressure which reduces the pore spaces of the rock mass. Sedimentary rocks have porosity variation to the maximum extent. The value ranges from 1% to 90%. In fact, it depends on the age of formation.

The igneous rocks have porosity less than 1% to 2%. But due to weathering, the value goes as high as 20%. Hence, the measured value of porosity gives an idea of weathering in the rock mass also. Chalk is found to be the most porous rock. This value is found to be as high as 50% or more. The porosity of a rock material is a measure of its water-holding capacity also. This, sometimes, helps in evaluating the water yield of a stratum. More porous rock may not be suitable for an engineering structure.

6·4. Density

Density is defined as the mass per unit volume of the rock. Depending upon a requirement, the density may be expressed as dry density, bulk density or saturated density.

Dry density refers to mass per unit volume when the rock mass is completely dry *i.e.* void contain only air.

Bulk density refers to mass per unit volume in normal condition, which means that the rock mass may contain some liquid and some air in its pores.

Saturated density refers to mass per unit volume when the rock mass is fully saturated.

Since the rock mass is made of solid minerals, the specific gravity of the solids is the ratio of its density and unit weight of water. Similar to the case of a porosity, rocks have a wide range of density which of course depends upon the mineral constituents and the degree of compaction in addition to the depth at which it is existing. In general, dry density of the rock varies from 2·6 gm/cc to 2·8 gm/cc in normal cases. However, Table-6·1 gives an idea of dry density of common types of rocks.

Table 6·1

Dry Densities of some Typical Rocks

Rock	*Dry Density (gm/cc)*	*Rock*	*Dry Density (gm/cc)*
Granite	2·65	Quartz, mica schist	2·82
Diorite	2·85	Rhyolite	2·37
Gypsum	2·30	Basalt	2·77
Dense limestone	2·70	Shale	2·25 to 2·62 (varies with depth)
Marble	2·75	Coal	0·7 to 2·0 (varies with ash content)

Although an approximate inference is drawn about the strength of a rock with values of porosity and density, but these values do not give information about the nature of a bond among the mineral grains.

The relation between bulk density and dry density is given by Eq. 6·2

$$y_d = \frac{y}{1+m} \qquad \ldots(6·2)$$

where y_d = the dry density,

y = bulk density,

and m = the moisture content of the sample.

6·5. Moisture Content

The moisture content of a rock sample is defined as the ratio of weight of water in the voids to the weight of dry solids in the sample

i.e. $$m = \frac{W_w}{W_s}$$

where m = the moisture content.

W_w = the weight of water,

W_s = the weight of solids.

Natural moisture content of a rock sample is the moisture content of the sample when taken from ground due to excavation or boring.

The moisture content is determined by noting the loss in weight of the sample after drying it for 24 hours at a temperature ranging from 105°C to 110°C. An excess natural moisture content gives an indication that the rock is more porous, making it of lesser strength. When confining pressure is increased around the rock mass, there is a decrease in moisture content making a rock mass stronger and *vice-versa*. That is why, due to an excavation near a rock mass,

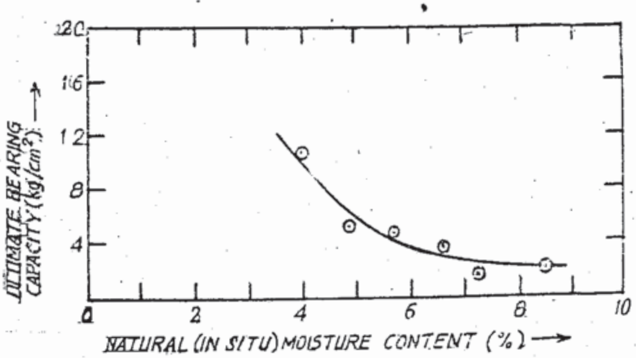


Fig. 6·1

confining stress is decreased which may cause the natural moisture content to increase resulting in a decrease in strength. Duncan (1969) has given a relation between the moisture content and the bearing capacity for a site in red mark. From the figure, it is quite apparent that bearing capacity decreases with increase in natural moisture content.

6·6. Degree of Saturation

Degree of saturation is defined as the volume of water in the void to the total volume of voids in the rock sample

i.e.
$$S=\frac{V_w}{V_v} \qquad \ldots(6\text{·}3)$$

where S=the degree of saturation,

V_w=the volume of water,

and V_v=the volume of void.

The rock mass having higher porosity has higher degree of saturation, if it lies below water table.

6·7. Permeability

6·7·1. Definition

Permeability refers to the ability of a porous material to allow a liquid to pass through its pores. Since the pores are connected with each other, the flow of a liquid takes place through the pores if there is difference in head at the two ends of the sample. Darcy has proposed the following equation for flow of liquid through a porous mass

$$Q \propto iA$$

or
$$Q = K\, i\, A \qquad \ldots(6{\cdot}4)$$

where Q = the discharge through the area A.

i = the hydraulic gradient

and K = the constant of proportionality which is known as the coefficient of permeability.

In general, from the term "coefficient of permeability" the coefficient word is dropped and only 'permeability' is commonly used. Hence, instead of "coefficient of permeability" of a rock mass we call 'permeability' of a rock mass. Although, the theory applies to flow of any liquid through the pores, for engineering purpose flow of water through pores are only considered, because it is related with actual problems encountered in the field.

Rewriting the equation (6·4) we have

$$K\, i = \frac{Q}{A} = v \qquad \ldots(6{\cdot}5)$$

Since i is dimensionless, the coefficient of permeability K, has unit of velocity. Generally it is expressed as cm per second or metre per day. The coefficient of permeability depends on the type a rock, pore size, entrapped air in the pores, temperature of the rock mass and viscosity of water which, in turn, again depends upon temperature of the flowing water.

6·7·2. Use of Permeability

In general, an engineer is interested in the permeability of the in-situ rock mass than in the permeability of rock material. The in-situ permeability takes into account the effect of fractures, bedding planes, joints and faults existing within the in-situ rock mass. Terzaghi has defined the permeability of rock material and in-situ rock mass separately.

The ability of flow through a rock material, including flow through macrofiractures, and some of the microfractures *is known* as "primary permeability" or primary percolations. whereas for flow through microfractures and microfractures of in-situ rock mass, "secondary permeability" or "secondary percolation" is used. But it is more suitable to use the term "secondary percolation."

Laboratory specimens of dense rock like granite, basalt, schist and cystalline limestone have very low values of permeability. Yet field tests in such rocks show higher values of permeability due to regular sets of open joints and fractures in the rock mass.

Table 6·2 gives an idea about the permeability of Laboratory specimen and Field test of some typical types of rocks.

Table 6·2
Permeability of Typical Rocks

Type of Rock	*K (cm/sec)*	
	Laboratory Sample	*Field*
Sandstones	3×10^{-3} to 8×10^{-8}	1×10^{-3} to 3×10^{-8}
Shale	1×10^{-9} to 5×10^{-13}	1×10^{-8} to 1×10^{-11}
Limestone dolomite	1×10^{-5} to 1×10^{-13}	1×10^{-3} to 1×10^{-7}
Basalt	1×10^{-12}	1×10^{-2} to 1×10^{-7}
Granite	1×10^{-7} to 1×10^{-11}	1×10^{-4} to 1×10^{-9}
Schist	1×10^{-8}	2×10^{-7}

6·7·3. Experimental Determination of Permeability

Experimental determination of permeability of a rock laboratory sample can be done by the longitudinal percolation tests or the Radial percolation tests. But radial percolation tests are more suitable.

6·7·3·1. Longitudinal Test

A cylindrical sample of the rock is covered with a plastic coating

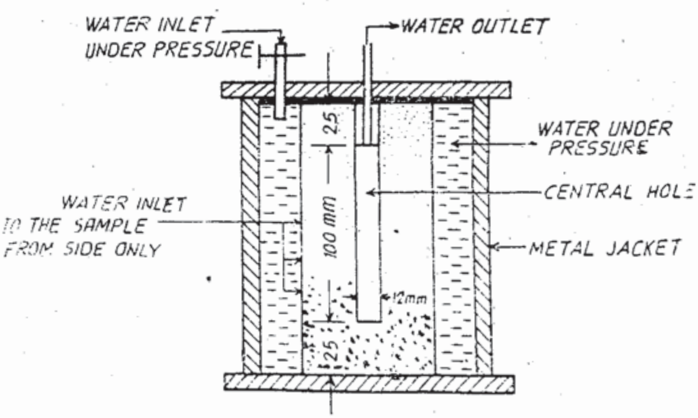


Fig. 6·2

on the vertical face. The sample, with plastic coating, is put in a chamber containing water as shown in Fig. 6·2.

Pressure through water is applied at one end while from other end, seeping water is collected and amount of discharge is known. From Eq. (6·6) the coefficient of permeability can be calculated

$$K=\frac{Q\,L}{A.ht} \qquad \ldots(6\cdot6)$$

where Q=the discharge collected in time t

h=pressure head applied for the measurement of permeability

A=the area of the sample.

6·7·3·2. Radial Test

In the core sample of diameter 60 mm and length about 150 mm, a hole is made at the centre of the specimen of diameter 12 mm and length 125 mm as shown in Fig. 6·3. The sample is enclosed in a metal cylinder.

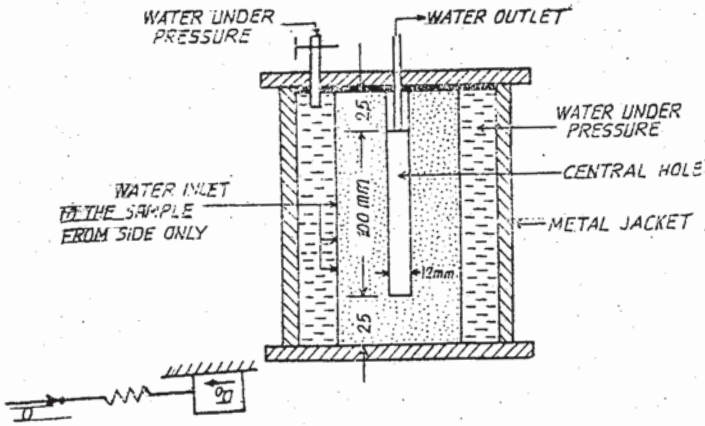


Fig. 6·3

The pressure from outside periphery of the cylinder is applied due to which a flow takes place and collected in the central hole, where the pressure is atmospheric. Let L be the length of the inside hole and outside and inside of radii of the hole be R_2 and R_1 respectively. Therefore flow through a cylindrical surface with a radius r is

$$q=K.Ai$$
$$=K.\,2\pi rL\left(\frac{dp}{dr}\right)$$

If Q be the total discharge

$$\therefore \qquad \int q = \int K 2\pi r L \left(\frac{dp}{dr} \right)$$

or
$$\int \frac{dr}{r} = \int \frac{K 2\pi L}{q} \, dp$$

Now, integrating within limit R_2 and R_1

$$\log_e \frac{R_2}{R_1} = K \frac{2\pi L}{Q} p$$

$$K = \frac{Q}{2\pi L p} \log_e \frac{R_2}{R_1}$$

$$= \frac{2 \cdot 3 \, Q}{2\pi L p} \log_{10} \frac{R_2}{R_1} \qquad \ldots(6 \cdot 7)$$

Since the discharge Q has been noted in time t, the Eq. (6·7) has to be divided by t seconds

$$\therefore \qquad K = \frac{2 \cdot 3 \, Q}{2\pi L p t} \log_{10} \frac{R_2}{R_1} \qquad \ldots(6 \cdot 8)$$

It is to be noted that the discharge Q should be in cc, length in cm, and pressure in cm head of water in order to get the permeability in cm/sec.

While conducting test, care should be taken that the applied pressure should not be too high or too low.

Above test gives the value of permeability of laboratory samples only. For ascertaining the permeability of the stratum with fractures and other discontinuities, pumping test is done which is costly. But considering the cost of a project for which this information is required the involved cost might not be much when compared with the total cost of the project.

6·8. Electrical Properties

Electrical properties are of interest mostly in a geolcgical prospecting where electrical resistivity methods are used. Most of the rocks are dielectrics and hence measurements of dielectric constants are done for interpretation of the data. This property is also of importance in prospecting for ground water recources because due to the presence of water in the pores of the rock, the dielectric properties of the rocks change sharply and hence interpretation of the data is done for the location of ground water reservoirs.

6·9. Thermal Properties

Thermal properties are of more importance to the engineers, specially for tunnels and other underground openings such an underground power houses etc. Increase in temperature of the rock or a frequent change in the rock temperature makes it weaker due to the formation of cracks in the rock mass. Hence, a knowledge of thermal conductivity and coefficient of thermal expansion and contraction of rock mass is essential. Thermal conductivity K

$$K = \frac{Qx}{At \, (T_2 - T_1)} \qquad \ldots(6 \cdot 9)$$

where Q=amount of heat transferred in perpendicular direction through an area A

T_2-T_1=temperature difference in °C between two points at a distance x.

Temperature changes induce thermal stresses causing thermal strains in the rock mass. Increase in length due to a change in temperature can be known if the coefficient of thermal expansion is known for different types of rocks. Change in length and volume can be known with known values of coefficient of linear thermal expansion and volumetric thermal expansion.

$$\triangle L=\alpha L \triangle T \quad \ldots(6{\cdot}10)$$

where α=coeff. of linear thermal expansion,

L=length of specimen,

and $\triangle T$=change in temperature.

Knowing the changed length change in volume also can be known. Table 6·3 gives the values of coefficient of linear thermal expansion of some typical rock.

Table 6·3

"Coefficient of Thermal Expansion of some Rocks"

Name of Rock	*Coeff. of linear thermal expansion per °C*
Igneous	
Granites	34×10^{-7} to 66×10^{-7}
Basalts	22×10^{-7} to 35×10^{-7}
Diabase	31×10^{-7} to 35×10^{-7}
Gabbro	20×10^{-7} to 30×10^{-7}
Sedimentary	
Limestones and dolomites	24×10^{-7} to 68×10^{-7}
Sandstones	36×10^{-7} to 65×10^{-7}
Metamorphic	
Gneisses	34×10^{-7} to 44×10^{-7}
Marbles	34×10^{-7} to 51×10^{-7}
Quartzites	60×10^{-7} to 61×10^{-7}
Schists (crystalline)	34×10^{-7} to 43×10^{-7}
Slates	45×10^{-7} to 49×10^{-7}

Ref: *Griffith (1936).*

6·10. Swelling

Swelling in general is defined as an increase in volume of the mass due to suction of water or due to contact of water for a longer time. When there is an excavation in the rock mass, the confining stress reduces at that site, which causes an increase in suction pressure at the other sides and this causes entry of water into the pore causing an increase in volume. This increase in volume, sometimes, causes cracks inside the rock mass. Again, a suction pressure causes sometimes, splitting in the sedimentary rocks at the bedding plane, if the cementing force is not sufficient. Swelling is found to be more in weaker type of rocks. Hence, swelling is related with unconfined compression strength of the rock. With increasing value of unconfined compressive strength, the tendency of swelling reduces. Relationship between degree of saturation and swelling potential in a rock mass is shown in Fig. 6·4 Duncan (1969) with the help of which relation and between water content swelling potension can be inferred.

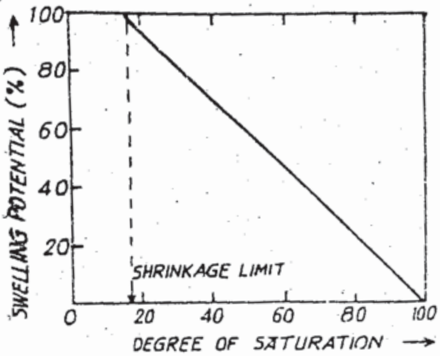


Fig. 6·4

As moisture content increases beyond the shrinkage limit, swelling potential increases till rock is saturated. Beyond the saturation limit, swelling potential is zero. Hence, before making an excavation for underground work swelling study is also necessary for the rock masses. For determination of swelling characteristics, generally a dry cylindrical sample is allowed to soak water and a corresponding change is noted with the help of which swelling characteristics of the rock sample is known. If the sample remains as a coherent block, with no significant swelling with an absorption of water, it is known as "cemented" and if it remains as a coherent block with significant swelling on absorption of water, the sample will be known as compacted. The degree of cementation governs the ability of individual grains to reorient under tensile stresses due to water. Clays, shales, marls and mudstones have the greatest tendency for swelling.

6·11. Anisotropy

Due to sequence in formation, a rock mass may not be isotropic. It may be anistropic due to existence of bedding planes. An isotropic material, like concrete or steel, has the same property in three axes. But anisotropic material has some weakness in a particular direction or axis. Specially, in sedimentary rocks, degree of anisotropy is more. Due to anisotropy, a rock mass has different properties along the plane of weakness. Hence it is necessary to know the plane of weakness. This can be done by taking samples from a rock mass in all the three perpendicular directions. A core of 50 mm diametre and 100 mm length is taken and discs of 25 mm thickness are cut out of this core. Thus there will be three sets of discs for three different axes. Now discs of each set are kept separately, and tested under a compression testing machine under two point loads. An average value of three sets are calculated. If the rock mass is having anisotropy, one set of the discs will be having less strength, and the axes of weakness, thus, can be known.

6·12. Durability

Durability may be defined as a resistance to distruction. If a rock mass is more durable, it will last for a longer period when put to an use. Durability of rock mass will depend on the nature of environment against which it is going to be used.

Table 6·4

Durability Classification

Group	*% Retained after one 10 minute cycle*	*% Retained after two 10 minutes cycles*
Very high durability	>99	>98
High durability	98−99	95−98
Medium high durability	95−98	85−95
Medium durability	85−95	60−85
Low durability	60−85	30−60
Very Low durability	<60	<30

In other words, the durability depends on climate and atmosphere and the amount of exposure of rock mass. Agencies, which reduce the durability of rocks, are hydration, slaking, solution. oxidation, abrasion etc. These may act alone and in combination. In some cases, a rock mass starts changing as soon as a fresh surface is exposed to the atmosphere. Of course the process of decay retards with time. Hence, before using a rock mass, its degree of decay or its durability has to be ascertained. To describe the ranking of a rock durability, an index to alteration is commonly used and it is known as slake durability index.

Gamble (1971) has proposed a test to determine slake durability Index (I_d). 500 gm of dry rock is broken into 10 pieces and put in a drum of 140 mm dia and 100 mm length. The cylindrical wall of the drum is made of sieve mesh of 2 mm openings. The drum is turned in a water bath for ten minutes at a rate of 20 revolutions per minute. The sample in the drum is dried and again rotated for 10 minutes in the drum at the rate of 20 revolutions per minute. The sample of the drum is collected and dried. The percent of rock retained in the drum (on dry weight basis) is known as slake durability Index (I_d). Franklin and Chandra (1972) recommend for only one cycle of revolution for 10 minutes and report the value of I_d. Table 6·4 gives the classification based on I_d value.

6·13. Mechanical Properties

Mechanical properties are also big described as Engineering Properties. These are the properties of rocks which help an engineer to fix the design parameters for a construction. Therefore, the most important property is the strength. Knowing the strength of a rock, a load to be superimposed on it can be determined. When load is applied to the rock the deformation should be within a limit and for this, deformability of the rock is to be known. A knowledge of Elastic and Plastic properties helps in analysing the performance of the structure after the imposition of load.

6·14. Strength

Strength is a general term. The ability of a material to resist an externally applied load is known as its strength. But, in rock mechanics, strength may be defined as the force per unit area required to bring about rupture in a rock mass at given environmental conditions. Environment is very important in considering the strength of a rock mass. A cylindrical rock sample when tested in unconfined compression *i.e.* without lateral restraint will behave as a brittle material and fail at comparatively lesser strain. But the same sample when tested under high lateral pressure behaves like a ductile material and fails at an increased strain. The increased confining pressure causes greater density of packing and consequently a higher resistance either in the intergranular bond or between crystals and grains. When a material is subjected to stresses acting for a long period of time, creep occurs. Elastic deformation of the rock material occurs initially on the application of load followed by a non-elastic flow causing rearrangement of the atomic structure of the rock material. On removal of the applied load, the creep strain is not completely recoverable. Recoverable strain depends on the magnitude of stress and time for which it has been applied. As fracture approaches, the rate of creep increases due to which microcracks are formed in the material. If the material is unconfined, propagation of these microcracks will be quicker whereas microfracture may be delayed or eliminated due to high confining pressures. Thus, we see that a consideration of environment of a rock mass is very important to be considered while discussing strength. At the time of determination of strength of a rock mass, laboratory specimen are to be prepared if not tested in "in-situ" condition. At the time of laboratory testing, the testing should be done with the same type of stress and environment under which the rock mass is going to be used.

In addition to the environment, rock strength depends on the following factors also.

(*a*) Size of the rock specimen,
(*b*) Type of test,
(*c*) Duration of test *i.e.* rate of loading,
(*d*) Loading condition,
(*e*) Cycle of loading,
(*f*) Confining pressure,
(*g*) Degree of saturation.

In addition to these factors, strength, of course, depends also upon physical characteristics of the rock materials.

6·14·1. Classification of Strength

Depending upon the type of loading and the stresses, the strength in general may be classified as

(*i*) Compressive strength, (*ii*) Tensile strength,
(*iii*) Shear strength.

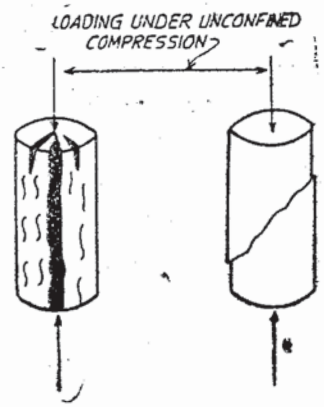
6·14·2. Compressive Strength

Compressive strength may be of two types :

(*i*) Uniaxial or unconfined compressive strength
(*ii*) Triaxial compressive strength.

6·14·2·1. Uniaxial Unconfined Compressive Strength

Uniaxial or unconfined compressive strength is the strength of the rock when a load on the rock acts in one direction only. There is no load along an axis perpendicular to the loading axis. The loading condition simulates the condition of pillars supporting the mine roof in case of underground mines. It already has been discussed that the unconfined compressive strength depends on size and length/dia. ratio of the sample and hence the value should be obtained in-situ condition as described in an earlier chapter. But when laboratory samples are being tested for required values of compressive strength, the rate of loading and the end conditions of loading are the two important factors which have to be kept in mind. Less time of testing shows a higher value of an unconfined compressive strength. If the friction between loading platen and the rock sample is more, the specimen will fail in shear, whereas in case of a smooth contact between



(*a*) Smooth contact between plates and ends of the specimen.
(*b*) Rough contact between plates and ends of the specimen.

Fig. 6·5

the loading platen and the sample, the sample will fail in tension. Vertical failure cracks will appear when the sample fails as shown in figure. Dry sample have high compressive strength and saturated samples have low values. Values of unconfined compressive strength of some of the common rocks are shown in Table 6·5.

Table 6·5

Compressive Strength of Various Rocks

Type of Rock	*Rock Name*	*Dry Unconfined Compressive Strength kg/cm²*
Igneous	Basalt	1500—3000
	Diabase	1200—2500
	Gabbro	1800—3000
	Granite	1000—2500
Sedimentary	Dolomite	800—2500
	Limestone	300—2500
	Sandstone	200—1700
	Shale	100—1000
Metamorphic	Gneiss	800—2000
	Marble	1000—2000
	Quartzite	870—3600
	Slate	1000—2000

6·14·2·2. Triaxial Compressive Strength

When a rock mass is subjected to an all-round pressure and if is further subjected to an additional vertical pressure, then strength exhibited by the rock mass is known as a triaxial compressive strength. The lateral pressure acting on the rock mass is also, sometimes, known as hydrostatic pressure and additional pressure which causes the failure in the mass is known as deviatric stress. In a laboratory, strength is evaluated by Triaxial Compression test. A sample of L/D ratio varying from 2 to 2·5 is kept in a chamber in which fluid pressure is applied in all the three directions. Through plunger vertical load is applied (as shown in Fig. 6·6) which causes failure in the sample. The lateral fluid pressure correspond to all-round pressure existing in the rock mass in the field. This test also helps in the determination of shear strength parameters of the rock material. Three cylindrical samples of the same rock material is subjected to three different chamber pressures. For each chamber pressure, the sample is loaded to failure. Thus, major and minor principal stresses are known. The Mohr's circles are drawn with three sets of observations. Mohr's envelope gives the value of cohesion "C" and the angle of internal friction "ϕ" as shown in Fig. 6·7.

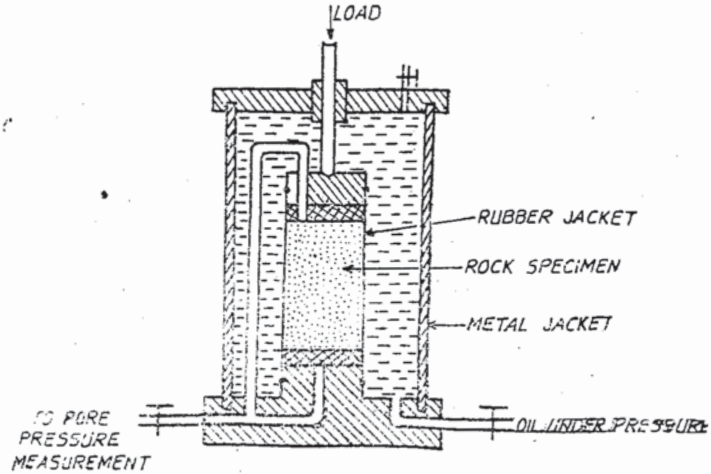


Fig. 6·6

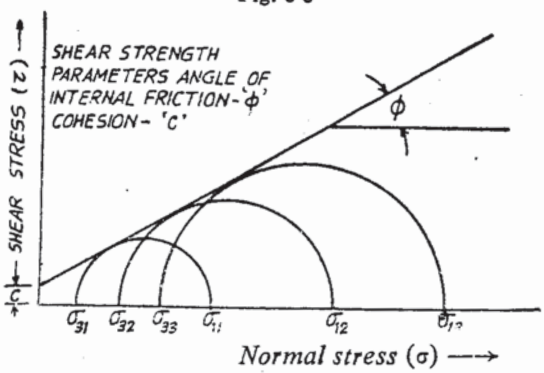


Fig. 6·7

If the sample is having moisture in the pores, then the effective stress parameters have to be known. This can be known when the Mohr's circle will be drawn with respect to an effective stress.

Effective stress = Total stress − Pore water pressure

Pore water pressure can be known during the triaxial test with attachments provided for the purpose.

6·14·3. Tensile Strength

The tensile strength of a material is defined as the maximum tensile stress which a material is capable of developing. In nature, rock mass is rarely subjected to direct tension, but it is subjected to tensile stresses. In the roofs and domes of underground openings, tension develops in the tensile zone of the rock mass. Tensile stresses also develop on the underside of a rock slab or a beam subjected to bending. Hence, a knowledge of tensile strength of the rock mass is also necessary. Rocks are weak in tension. It has been found that rocks possess tensile strength which is about 10% of its compressive strength. Since it is difficult to prepare rock samples for direct tensile tests, tensile strength of rock samples are determined indirectly by Brazilian tests or bending tests as described in chapter 7.

Reichmath (1963) described a point load test for the determination of tensile strength of rock samples. Cylindrical core specimen is kept such that its axis is horizontal. Compressive point loads are applied on the curved surface of the specimen by a compression testing machine through small diameter hard steel rollers at right angles to the axis of specimen. This loading produces tensile stresses perpendicular to the axis of loading and the tensile strength σ_t is given by the relation

$$\sigma_t = 0{\cdot}0675 \frac{P}{D^2} \qquad \ldots(6{\cdot}11)$$

where P = the failure load in kg
and D = the core diameter in cm.

Miller (1965) gave a relation between tensile strength σ_t and uniaxial compressive strength σ_u

$$\sigma_u = 21\ \sigma_t + 4000\ \text{lb/in}^2 \qquad \ldots(6{\cdot}12)$$

Typical values of tensile strength of some of the common rocks are given in Table 6·6.

Table 6·6. Tensile Strength of Various Rocks

Type of Rock	*Name of Rock*	*Tensile Strength σ_t kg/cm²*
Igneous Rocks	Basalt	60–120
	Diabase	60—130
	Gabbro	50—80
	Granite	40—80
Sedimentary Rocks	Dolomite	25—60
	Limestone	10—70
	Sandstone	20
	Shale	20—100
Metamorphic Rocks	Gneiss	40—70
	Marble	50—80
	Quartzite	30—50
	Slate	70—200

6·14·4·1. Shear Strength

Shear strength of rock may be defined as the capacity of a rock mass to take a shear stress. Shear strength may also be defined as the maximum resistance to deformation due to shear displacement caused by a shear stress.

Shear strength in a rock mass is derived from the surface frictional resistance along the sliding plane, interlocking between the individual rock grains and cohesion in the sliding surface of the rock. The pattern of joints, shear zones and faults in a rock mass reduces the effective shear strength of a rock mass. Therefore the shear strength of in-situ rock is highly anisotropic. Shear strength in a direction parallel to the discontinuities are much less than the intact rack mass. Hence, specially when the rock mass is supporting a concrete dam foundation, which is likely to experience a sliding force at the base due to an excessive water pressure, it is necessary to check the sliding resistance and shear resistance of the rock mass along the direction in which the maximum stress is expected to develop. The important problems in rock mechanics where a knowledge of shear strength of rock mass is needed, are the stability of rock slopes, stability of structure against sliding and stability problems of underground openings. Methods of determination of the in-situ shear strength and the laboratory method for determination of shear strength of rock samples have been discussed in other Chapters. Typical values of shear strength, angle of internal friction ϕ and coefficient of static friction of some common rocks are given in Table 6·7.

Table 6·7. Shear Strength, Angle of Fric. and Coeff. of Friction

Type of Rock	*Name*	*Shear Strength* kg/cm^2	*Angles of friction* $\phi°$	*Coeff. of static friction* $f = \tan\phi$
Igneous	Basalt	50—130	48—50	1·11—1·19
	Diabase	60—100	50—55	1·19—1·43
	Gabbro	40—85	10—31	0·18—0·60
	Granite	50—100	45—60	1·00—1·73
Sedimentary	Dolomite	25—70	22	0·40
	Limestone	15—70	35—50	0·70—1·20
	Sandstone	30	27—34	0·51—0·68
	Shale	30—110	15—30	0·27—0·58
Metamorphic	Gneiss	30—70	31—35	0·60—0·70
	Marble	35—80	32—37	0·62—0·75
	Quartzite	—	25·6—60	0·48—1·73
	Schist	—	62·25	1·90

Ref. Farmer (1968).

6·14·4·2. Point Load Strength

Since the determination of compressive strength takes time in a laboratory, and there are many factors on which the value depends, hence for quick estimation of an unconfined compressive strength of rock, point load strength is popular in rock mechanics. In the test, a rock specimen is loaded between two cones of hardened steel. Failure is caused by development of tensile cracks parallel to the loading axis. Point load strength I_s, as suggested by Brock and Franklin (1972) is given by equation 6·13.

$$I_s=\frac{P}{D^2} \qquad \ldots(6{\cdot}13)$$

where P=the load at rupture,

D=the distance between the point loads.

Tests are generally done on drilled core of length at least 1·4 times the diameter. The point load index $I_{s(50)}$ is reported as the point load strength of a 50 mm core. A common correlation between point load index and unconfined compressive strength is given by equation 6·14.

$$q_u=24\, I_{s(50)} \qquad \ldots(6{\cdot}14)$$

where q_u=the unconfined compressive strength of cylinders with length to diameter ratio 2 to 1

and $I_{s(50)}$=the point load strength corrected to a diameter of 50 mm.

Point load strength have been found to decrease by a factor 2 to 3 as core diameter increases from 10 mm to 70 mm.

Since it is very easy and quick to determine the point load strength, the value is found out at the time of boring itself and the value shown in the drill log./

6·14·4·3. Scale Effect on the Strength of Rocks

Due to presence of microfractures and discontinuity in rock mass generally more number of samples are tested for the mean value. Again, it has been noted that a larged sized sample shows lower strength than a smaller one. Let result of n number of tests be $x_1, x_2, x_3, \ldots x_n$. Then an average value is

$$M=\frac{x_1+x_2+x_3+\ldots x_n}{n}$$

Let d_i be the difference between the test result x_i and the mean M. Then by statistical theory, standard deviation

$$d_i=M-x_i$$

and

$$S_d=\sqrt{\frac{[d_i]^2}{(n-1)}} \qquad \ldots(6{\cdot}15)$$

The standard deviation for tensile, shear or compressive strength tests for rock is seen to be high.

To account for a size effect, Bernaix has proposed the ratio R_{10}/R_{60} where R_{10} and R_{60} are mean crushing strength of 10 mm dia. and 60 mm dia. cylinders respectively. Bernaix conducted tests on various types of rocks and the results are shown in Table 6·8. S_d/M, and R_{10}/R_{60} values give an idea for classification of rocks.

Table 6·8
Fissures and Crushing Strength of Rock

Rock type	*Fissures*	S_d/M	R_{10}/R_{60}
Very poor gneiss	Microfissures ; microfractures, very intense	0·37	2·90
Poor gneiss	Microfissures ; microfractures, microfractures, intense	0·30	1·90
Jurassic limestone	Microfissures, very few ; microfractures, intense	0·25	1·40
Biotite gneiss	Microfissures, average	0·22	1·25
Compact limestone	No microfissures	0·005	1·0

6·15. Elasticity — Hockin law, Elastic Material

If an external force, producing deformation does not exceed a certain limit, the deformation disappears with the removal of the force. The property of the material to recover the deformation is known as elasticity. The "limit" of deformation up to which the material is elastic is known as an elastic limit. The linear relation between the stress and deformation *i.e.* strain is known as Hooke's law. The strain within a proportional limit is given by equation 6·16 which also is known as Hooke's Law.

$$\epsilon=\frac{\sigma}{E} \qquad \ldots(6{\cdot}16)$$

where σ = the stress applied on the material

ϵ = strain in the material due to stress σ

and E is known as "modulus of elasticity" of the material.

The modulus of elasticity gives an idea about the elastic property of a material or elasticity of the material. Hence, if the material is to be stressed within an elastic limit, then noting the strain due to a loading, the value of existing stress can be calculated, if the value of E of the material is known. For equation 6·16 to be valid, it is necessary that the material should be isotropic and homogeneous. So long as the geometrical dimensions defining the form of a body are very large in comparison with the dimensions of a single crystal, the assumption of homogeneity can be used with an accuracy and if the crystals are oriented at random, the material can be treated as isotropic.

Many fresh and hard rocks are elastic when considered as laboratory specimens. But in "in-situ" state where the rock contains fissures, fractures, bedding planes and zones of altered rock and clay with plastic properties, most of the rocks do not exhibit perfect elasticity. Thus, in practice the property of elasticity of rocks depends upon their continuity, homogeneity and isotropy. That is why, for important large engineering projects, "in-situ" tests are done to find out stress strain property of the rock and then the theory of elasticity is applied, if found necessary. Sometimes theory of elasticity is applied to the rock mass with some suitable simplifying assumptions. Modulus of elasticity of rocks depends upon a rock type, its porosity, grain size and water content. Value of modulus of elasticity can be determined by static or dynamic methods. Static methods are more common for laboratory testing whereas dynamic methods are more popular for "in-situ" determination of E. But both methods can be applied for either cases. Higher values of modulus of elasticity indicates good quality of rock having sound composition. Typical values of modulus of elasticity of some common rocks are given in Table 6·9.

Table 6·9. Modulus of Elasticity (E) and Poisson's Ratio (ν) of Some Rocks

Type of Rocks	*Name of Rocks*	*Young modulus of elasticity (E)* $kg/cm^2 \times 10^5$	*Average values of Poisson's Ratio* (ν)
Igneous rocks	Basalt	2·0—10·0	0·14 —0·25
	Diabase	3·0—9·0	0·125—0·25
	Gabbro	6·0—11·0	0·125—0·25
	Granite	2·6—7·0	0·125—0·25
	Syemite	6·0—8·0	0·25
Sedimentary rocks	Dolomite	2 —4·4	0·08—0·20
	Limestone	1·0—8·0	0·10—0·20
	Sandstone	0·5—8·6	0·066—0·125
	Shale	0·8—3·0	0·11—0·54
Metamorphic rocks	Gneiss	2·0—6·0	0·091—0·25
	Marble	6·0—9·0	0·25—0·38
	Quartzite	2·6—10·2	0·23
	Schist	4·1—7·2	0·01—0·20

6·16. Plasticity — Confident.

Plasticity is defined as a property of the solid material to deform continuously and permanently without rupture under a stress exceeding the yield value of the material. Thus, the plasticity deals with the property of the material when stress has exceeded the yield value. Brittle materials follow elastic property and there is a collapse after elastic limit ; but ductile material follow plastic law above the elastic limit. At an ordinary temperature and pressure, rocks behave elastically, but in an environment at a higher pressure and temperature plastic deformations of rocks take place. In a plastic state permanent deformation may occur without fracture. In a rock material, elastic deformation occurs on initial application of load followed by a non-elastic flow due to re-arrangement of the atomic structure of the rock material. On further application of the load, plastic flow takes place followed by formation of microcracks within rock material. If high confining pressure acts, then on-set of fracture may be delayed or eliminated. The phenomenon of creep in the rock material is due to plastic flow.

Example

6·17. Poisson's Ratio

When a material is subjected to uniaxial compressive stress σ_x strain in x direction is given by

$$\epsilon_x = \frac{\sigma_x}{E} .$$

But, due to shortening of the material in the x-direction there is corresponding increase in dimensions of the material in lateral dimensions. If the sample is cylindrical, then the lateral strain is given by equation

$$\begin{aligned} \epsilon_y &= \epsilon_z \\ &= \nu \frac{\sigma_x}{E} \\ &= \nu \epsilon_x \qquad \ldots(6{\cdot}17) \end{aligned}$$

In equation 6·17, ν is a constant called Poisson's ratio. Hence, if longitudinal strain is known lateral strain can be evaluated with known value of Poisson's ratio of the material. Poisson's ratio

$$\nu = \frac{\epsilon_{lat}}{\epsilon_{log}}$$

where ϵ_{log} = strain parallel to the direction of applied load

σ_{lat} = strain at right angle to the direction of applied load.

The reciprocal of Poisson's ratio is known as Poisson's number and is denoted by m.

Presence of cracks decreases the value of Poisson's ratio, but if the cracks are oriented parallel to the direction of application of the load, they tend to open up, causing a higher lateral strain and, thus a higher value of Poisson's ratio is obtained. (Due to propagation of cracks at higher stresses the value of Poisson's ratio is also higher. Hence for laboratory determination of Poisson ratio, the stress level should be kept the same as expected in the 'in-situ' condition. Due to confining pressure, Poisson's ratio is lowered in weaker rocks. The methods of evaluation of Poisson's ratio have been discussed in earlier chapter. Average value of Poisson's ratio of some of the common rocks are given in Table 6·9.

Because of fissures and fractures, the effective modulus of rock mass E_{eff} is not equal to the modulus of E of rock material. Similarly, the effective Poisson's ratio ν_{eff} for rock mass is different for the ν measured value of a rock material. On the suggestion of Waldorf et al (1963), Jaeger (1966) has evolved the Eqn. 6·18 for evaluation of E_{eff} or ν_{eff}. In their model, a rock mass is considered to be cut into parallelepipeds by a series of horizontal fractures with two other series of vertical fractures at right angles to each other. The load being applied in a vertical direction. The equation obtained by Jaeger is

$$\frac{E_{eff}}{E}=\frac{\nu}{\nu_{eff}}$$

$$=\nu\ \frac{(\sigma_1+\sigma_3)}{\sigma_3}. \qquad ...(6·18)$$

where σ_1 = vertical stress

σ_3 = horizontal stress

In the above equation, horizontal stress in the two horizontal axes are considered to be equal *i.e.*,

$$\sigma_2=\sigma_3.$$

6·18. Deformability

Deformability of a rock means the capacity of the rock to strain under applied loads, or in response to the removal of a load, in case of excavation. For a perfect performance of the structures made over the rock mass it is necessary to know how much deformation will take place in the rock mass under loading system. For example if a concrete dam rests on a rock mass, then a knowledge about the settlement at different points is necessary. The total settlement may not be excessive, but if the settlement at the different points are different then the dam body will be subjected to differential settlement due to which there may be cracks in the body of the dam when the differential settlement is not within tolerable limit. Similarly, if a pressure tunnel is being designed, a knowledge of expansion of the lining under an operating pressure and the amount of recovery, when the pressure is lowered is essential. On the basis of this information lining can be designed properly with the required amount of steel needed.

Deformability of rock depends on the type of a rock, various rock defects, degree of weathering, type of load, state of stress within the rock mass as well as elastic and rheological properties of rock. Methods to determine deformability of a rock mass have been discussed in an earlier chapter. In this chapter, deformability of different types of rocks will be discussed. An idea of deformability of a rock mass can be obtained by a stress-strain curve. To study the different stages in a rock mass, while it is being stressed, a study, of an idealized stress-strain diagram for a ductile material, is necessary.

Fig. 6·8 shows such a diagram. Up to point Y the strain is elastic and the material is said to be linear elastic till it has been stressed up to point Y. Point LP is the limit of proportionality and

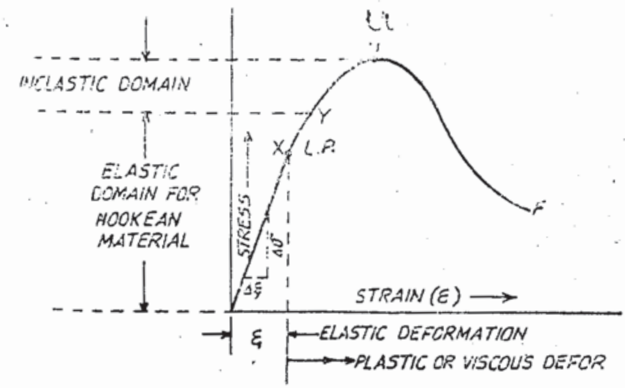


Fig. 6·8

for most of the materials point LP coincides with Y. Beyond point Y, the material has a ductile behaviour and the point Y at which the transition from elastic to ductile behaviour takes place, is known as the yield point and the corresponding stress is known as the yield stress. If the loading is continued beyond point U, failure of the material takes place. Stress at point U is known as the ultimate stress. On increasing the strain further beyond point U, the stress drops as shown by UF. In the range UF, the material behaves plastically. The behaviour of a material can be expressed sometimes by different elastic modulii. They are *initial tangent* modulus, tangent modulus and scant modulus.

Initial tangent modulus is the slope of the line which is tangent to the stress-strain curve at zero load. It is denoted by E_i.

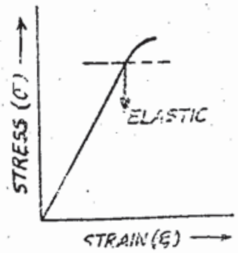
Tangent modulus is the slope of the line which is tangent to any given point on the stress-strain curve. It is denoted by E_t.

E_{t50} means the slope of the line which is tangent to the stress-strain curve at a point of 50% ultimate load.

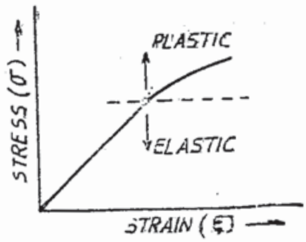
Scant modulus is the slope of the line which is joined by any point on the stress-strain curve with the origin.

High values of elastic modulus indicate a better strength of the material, and deformability of the material is less.

Miller (1965) has given stress-strain curves for different types of rocks. These results are based on uniaxial stress-strain curves. Curve of Fig. 6·9 exhibits nearly a straight line behaviour until a sudden failure occurs. Such type of behaviour is observed in basalts,

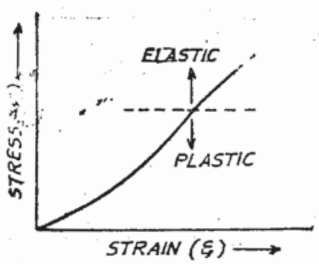


Plastic
Fig. 6·9

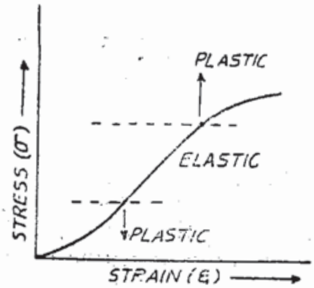


Elastic-Plastic
Fig. 6·10

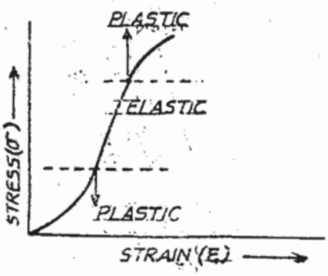
quartzite, diabase, dolomite and strong lime stones. Nature of the curve is elastic.



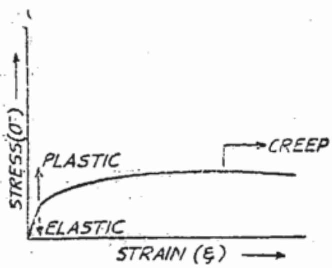
Plastic-Elastic
Fig. 6·11



Plastic-Elastic-Plastic
Fig. 6·12



Plastic-Elastic-Plastic
Fig. 6·13



Elastic-Plastic-Creep
Fig. 6·14

Curve of Fig. 6·10 shows a continuously inelastic yielding as the failure stress is approached. This type of behaviour is exhibited by softer lime-stones, silstone, tuff and other soft rocks. Nature of the curve is elastic-plastic.

Curve of Fig. 6·11 is exhibited by sandstones granites, schist and some diabases. The curve has got plastic-elastic nature.

S-shaped curve as shown in Fig. 6·12 is common in metamorphic rocks such as marbles and gneiss.

Schist samples cored perpendicular to the foliation show high compressibility as shown by Fig. 6·13. Nature of the curves are plastic-elastic-plastic.

Curve of Fig. 6·14 is shown by rock salt. It has a small initial straight line portion followed by increasing inelastic deformation and a continuous creep. Nature of the curve is elastic-plastic creep.

Curves of Figs. 6·11, 6·12 and 6·13 are concave upward initially, and then, they are concave downward. This is because of the presence of microcraks or foliation surface. After some loading, these microcracks close up and the rock becomes stiffer and finally the failure is of brittle type.

It is to be noted that a state of stress, influence the strength, stiffness, ductility and creep properties of the intact rock. As the confining stress increases, the ultimate failure also increase and the rock mass fails at higher strains. The deformability of rock mass is to be estimated by the deformation modulus. Method of estimation of deformation modulus has been explained in the earlier chapter.

6·19. Hardness

Hardness of rock is defined as its resistance is abrasion. This property helps in estimating rock decay when put to odd conditions. It also gives an idea of strength criterian of rocks. Hardness of rock depends upon the strength of chemical bonds. Although for metals Brinell's hardness test is recommended, for evaluation of hardness, for rock, Mohr's empirical hardness scale is used. The hardness is estimated by scratching the rock material. Table 6·10 describes the hardness of different minerals.

Table 6·10
Mohr's Scale of Hardness of Minerals

No. of relative hardness scale or rating H	*Mineral*	*Chemical composition*	*Remarks*
1	Talc	$Mg_3\ Si_4\ O_{10}\ (OH)_2$	Softest, can be scratched by finger nail.
2	Gypsum	$CaSO_4\ 2H_2O$	Can be scratched by finger nail.
3	Calcite	$CaCO_3$	A copper coin or a brass pin can scratch.
4	Fluorite	CaF_2	May be scratched by steel point.
5	Apatite	$Ca_5F(PO_4)_3$	Can be scratched by a knife.
6	Orthoclase (Feldspars)	$KAl\ Si_3\ O_8$	Can be scratched by a knife blade of good quality steel.
7	Quartz	SiO_2	Scratches steel and glass.
8	Topaz	$Al_2\ SiO_4\ (F,OH)_2$	Great hardness.
9	Corundum	Al_2O_3	Harder than any other natural mineral except diamond. An important abrasive.
10	Diamond	C	The hardest substance known. All diamonds are of the same hardness.